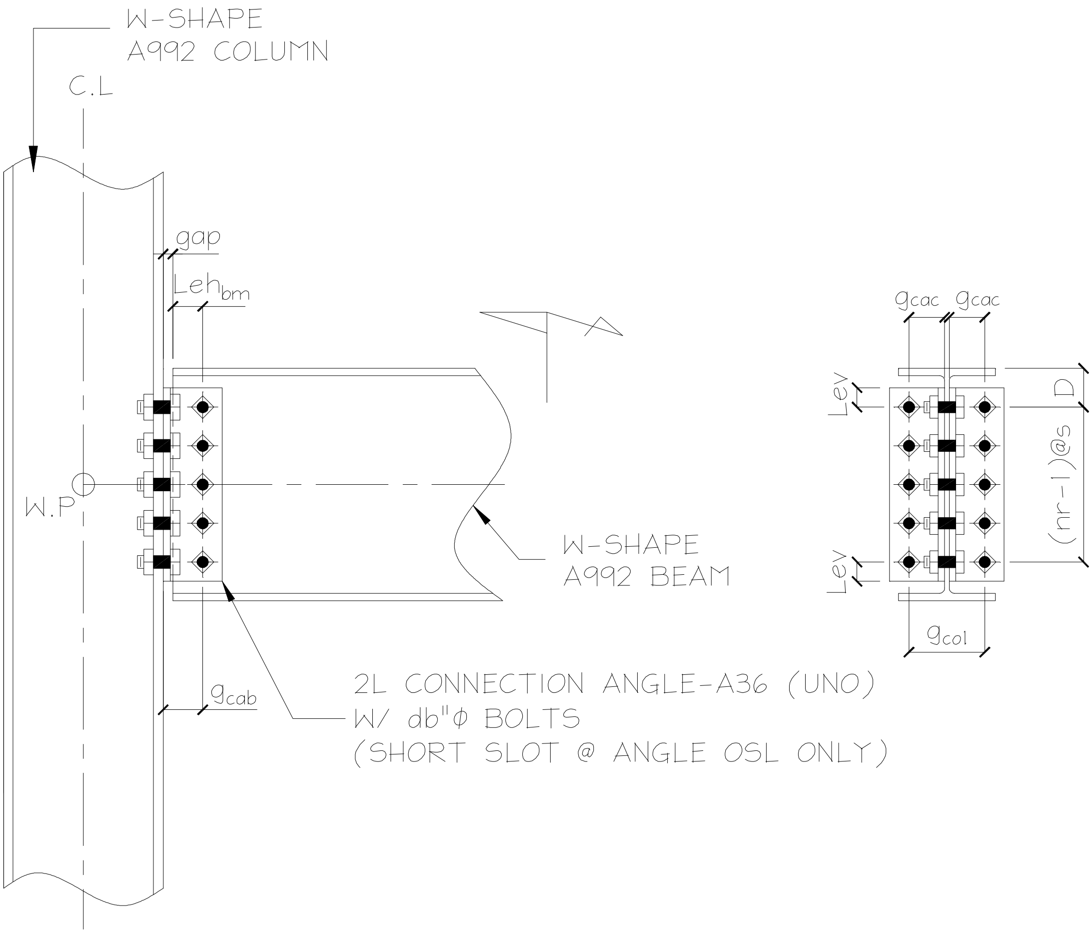
5404 Gurley Avenue
Dallas, TX 75223

SHEAR CONNECTION: DESIGN OF W-SHAPE BEAM TO W-SHAPE COLUMN FLANGE CLIP ANGLE CONNECTION (BOLTED-BOLTED)



NOTE: (FIGURE ABOVE DOES NOT REPRESENT ACTUAL DESIGN, REFER ON ATTACHED CONNECTION SCHEDULE FOR NUMBER OF BOLTS)

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I. DESIGN DATA AND LOAD (ASD - AISC 14th Edition)

COLUMN PROPERTIES (col): W12X96 - A992

$Fy_{col} := 50ksi$ $d_{col} = 12.7 \cdot in$ $tw_{col} = 0.55 \cdot in$ $k1_{col} = 1.125 \cdot in$

$Fu_{col} := 65ksi$ $bf_{col} = 12.2 \cdot in$ $tf_{col} = 0.9 \cdot in$ $k_{col} = 1.812 \cdot in$

$Ag_{col} = 28.2 \cdot in^2$ $Sx_{col} = 131 \cdot in^3$ *Column Gage,* $g_{col} := 5.5in$

BEAM PROPERTIES (bm): W18X50 - A992

$Fy_{bm} = 50 \cdot ksi$ $d_{bm} = 18 \cdot in$ $tw_{bm} = 0.355 \cdot in$ $k1_{bm} = 0.813 \cdot in$

$Fu_{bm} = 65 \cdot ksi$ $bf_{bm} = 7.5 \cdot in$ $tf_{bm} = 0.57 \cdot in$ $k_{bm} = 1.25 \cdot in$

$Ag_{bm} = 14.7 \cdot in^2$ $Sx_{bm} = 88.9 \cdot in^3$ *Length of beam,* $L_{bm} := 10ft + 0\frac{0}{0}in$

CONNECTION ANGLE PROPERTIES (ca): 2L4X4X3/8 - A36

$Fy_{ca} = 36 \cdot ksi$ $leg1_{ca} = 4 \cdot in$ $t_{ca} = 0.375 \cdot in$

$Fu_{ca} = 58 \cdot ksi$ $leg2_{ca} = 4 \cdot in$ *Beam side bolt gage:* $g_{cab} = 2.5 \cdot in$

Number of Connection Angle: $n_{ca} := 2$ *Column side bolt gage:* $g_{cac} = 2.572 \cdot in$

BOLTS:

Bolt Diameter, $db = 0.75 \cdot in$ Bolt_Type = "A325-N"

Bolt Shear Strength, $\Lambda rv = 11.928 \cdot kips$ Conn_type = "Bearing-type"

Bolt Tensile Strength, $\Lambda rn = 19.88 \cdot kips$

Gap between edge of beam to edge of support, $gap := \frac{1}{2} in$

Beam Edge Distance, $Leh_{bm} = 2 \cdot in$

Clip Angle Vertical Edge Distance, $Lev = 1.25 \cdot in$

Clip Angle Horizontal Edge Distance, $Leh_{cac} = 1.428 \cdot in$ $Leh_{cab} = 1.5 \cdot in$

Bolt Vertical Spacing, $s = 3 \cdot in$

Hole diameter:

Clip Angle (Beam side), $hd_{bcav} = 0.875 \cdot in$ $hd_{bcah} = 0.875 \cdot in$

Clip Angle (Column side), $hd_{ccav} = 0.875 \cdot in$ $hd_{ccah} = 1.063 \cdot in$

Beam, $hd_{bm} = 0.875 \cdot in$

Column, $hd_{col} = 0.875 \cdot in$

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Bolt Horizontal Spacing (For Multiple bolt lines), $sv = 3 \cdot in$

Bolt First Down from Top of beam, $D = 3 \cdot in$

number of bolt rows: $nr := 5$

number of vertical bolt lines: $nv := 1$

total number of bolts: $n := nr \cdot nv$ $n = 5$

SAFETY AND RESISTANCE FACTORS:

Safety Factor, Ω (ASD)

Resistance Factor, ϕ (LRFD)

Modification Factor, $\Lambda = \frac{1}{\Omega}$ (IF ASD) $\Lambda = \phi$ (IF LRFD)

	safety factor	*resistance factor*	*modification factor*
For member shear (C, WT, L)	$\Omega_v = 1.67$	$\phi_v = 0.90$	$\Lambda_v = 0.60$
For shear rupture,	$\Omega_{vr} = 2.00$	$\phi_{vr} = 0.75$	$\Lambda_{vr} = 0.50$
For shear yielding,	$\Omega_{vy} = 1.50$	$\phi_{vy} = 1.00$	$\Lambda_{vy} = 0.67$
For block shear,	$\Omega_{bs} = 2.00$	$\phi_{bs} = 0.75$	$\Lambda_{bs} = 0.50$
For member/bolt in bearing,	$\Omega_{brg} = 2.00$	$\phi_{brg} = 0.75$	$\Lambda_{brg} = 0.50$
For flexural local buckling / flexural strength,	$\Omega_b = 1.67$	$\phi_b = 0.9$	$\Lambda_b = 0.60$
For flexural rupture,	$\Omega_{fr} = 2.00$	$\phi_{fr} = 0.75$	$\Lambda_{fr} = 0.50$
For shear on N-type bolts,	$\Omega_{vtN} = 2.00$	$\phi_{vtN} = 0.75$	$\Lambda_{vtN} = 0.50$

APPLIED LOAD:

% UDL, $UDL := 0.5$

Given Load if any, $V_{giv} := 0kips$

Beam Shear Load, $V = 100.8 \cdot kips$ **50% UDL**

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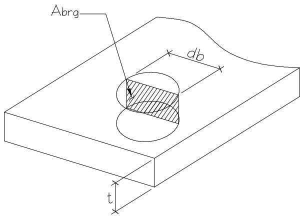
II. CALCULATIONS:

A. BEAM CHECK

1. Bolt Bearing Capacity on Beam

(AISC 14th Ed. Specifications Chapter J, Section J3.10, pages 16.1-127 to 16.1-128)



Bearing Area,

$Abrg_{bm} := db \cdot tw_{bm}$

$Abrg_{bm} = 0.266 \cdot in^2$

Allowable Bearing Strength using edge distance, (J3-6a, J3-6c)

$$Fbe := \Lambda_{brg} \cdot Fu_{bm} \cdot \begin{cases} \min\left[1.0 \cdot (D - 0.5 \cdot hd_{bm}) \cdot tw_{bm}, 2.0 \cdot Abrg_{bm}\right] & \text{if } hd_{bm} \geq hd_{ls} \\ \min\left[1.2 \cdot (D - 0.5 \cdot hd_{bm}) \cdot tw_{bm}, 2.4 \cdot Abrg_{bm}\right] & \text{otherwise} \end{cases}$$

$Fbe = 20.767 \cdot kips$

Allowable Bearing Strength using bolt spacing, (J3-6a, J3-6c)

$$Fbs := \Lambda_{brg} \cdot Fu_{bm} \cdot \begin{cases} \min\left[1.0 \cdot (s - hd_{bm}) \cdot tw_{bm}, 2.0 \cdot Abrg_{bm}\right] & \text{if } hd_{bm} \geq hd_{ls} \\ \min\left[1.2 \cdot (s - hd_{bm}) \cdot tw_{bm}, 2.4 \cdot Abrg_{bm}\right] & \text{otherwise} \end{cases}$$

$Fbs = 20.767 \cdot kips$

Bolt Bearing Capacity,

$Rbrg_{bm} := nv \cdot \left[\min(Fbe, n_{ca} \cdot \Lambda rv) + \min(Fbs, n_{ca} \cdot \Lambda rv)(nr - 1)\right]$

$Rbrg_{bm} = 103.837 \cdot kips$ $V = 100.8 \cdot kips$

RESULT = Bearing Capacity > Force Applied, OK

2. Shear Capacity of beam

(AISC 14th Ed, Specifications Chapter G, Section G2.1, pages 16.1-67 to 16.1-69)

Clear distance between flanges of beam, less the fillet or corner radii,

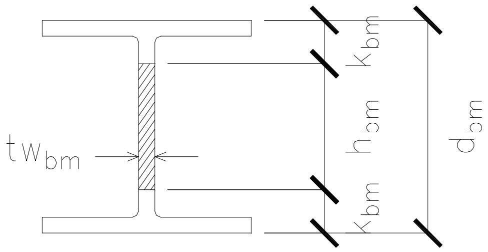
$h := d_{bm} - 2 \cdot kdes_{bm}$

$h = 16.056 \cdot in$

Limiting depth-thickness ratio,

$$h_{tw} := \frac{h}{tw_{bm}}$$

$$h_{tw} = 45.228$$



Clear distance between transverse stiffeners,

$$a := \begin{cases} 0in & \text{if } h_{tw} < 260 \\ \min\left[3 \cdot h, \left(\frac{260}{h_{tw}}\right)^2 \cdot h\right] & \text{otherwise} \end{cases}$$

$$a = 0 \cdot in$$

Web plate buckling coefficient,

$$kv := \begin{cases} 5 & \text{if } h_{tw} < 260 \\ 5 + \frac{5}{\left(\frac{a}{h}\right)^2} & \text{otherwise} \end{cases} \quad (G2\text{-}6)$$

$$kv = 5$$

Web shear coefficient,

$$Cv := \begin{cases} 1 & \text{if } h_{tw} \le 1.1 \cdot \sqrt{\frac{kv \cdot E}{Fy_{bm}}} \quad (G2\text{-}3) \\ \frac{1.1 \cdot \sqrt{\frac{kv \cdot E}{Fy_{bm}}}}{h_{tw}} & \text{if } 1.1 \cdot \sqrt{\frac{kv \cdot E}{Fy_{bm}}} < h_{tw} \le 1.37 \cdot \sqrt{\frac{kv \cdot E}{Fy_{bm}}} \quad (G2\text{-}4) \\ \frac{1.51 \cdot E \cdot kv}{h_{tw}^2 Fy_{bm}} & \text{if } 1.37 \cdot \sqrt{\frac{kv \cdot E}{Fy_{bm}}} < h_{tw} \quad (G2\text{-}5) \end{cases}$$

$$Cv = 1$$

Shear Capacity of Section,

$$Rv_{bm} := \Lambda_{vbm} \cdot 0.6 \cdot Fy_{bm} \cdot d_{bm} \cdot tw_{bm} \cdot Cv \quad (G2\text{-}1)$$

$Rv_{bm} = 127.8 \cdot kips$ $\qquad V = 100.8 \cdot kips$

RESULT = Shear Capacity of Section > Force Applied, OK

B. BEAM TO CLIP ANGLE CHECK

1. Bolt Shear Capacity

(AISC 14th Ed. Specs. Chapter J, Section J3.6, page 16.1-125)

Shear Capacity per Bolt,

$\Lambda rv = 11.928 \cdot kips$

Bolt Group Shear Capacity,

$Rb_{v1} := n_{ca} n \cdot \Lambda rv$

$Rb_{v1} = 119.282 \cdot kips$ $V = 100.8 \cdot kips$

RESULT = Bolt Shear Capacity > Force Applied, OK

2. Check for Spacing

(AISC 14th Ed. Specifications Chapter J, Section J3.3 and J3.5, pages 16.1-122 to 16.1-124)

Vertical Spacing,

$s = 3 \cdot in$

$s_{min} := 2\frac{2}{3} \cdot db$

$s_{min} = 2 \cdot in$

$s_{max} := min\left(12in, 24 \cdot min\left(tw_{bm}, t_{ca}, tf_{col}\right)\right)$

$s_{max} = 8.520 \cdot in$

RESULT = s > s.min & s < s.max, OK

Horizontal Spacing,

$sv = 3 \cdot in$

$sv_{min} := 2\frac{2}{3} \cdot db$

$sv_{min} = 2 \cdot in$

$sv_{max} := min\left(12in, 24 \cdot min\left(tw_{bm}, t_{ca}, tf_{col}\right)\right)$

$sv_{max} = 8.520 \cdot in$

RESULT = This check is not applicable

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3. Check for Edge Distance

(AISC 14th Ed. Specifications Chapter J, Section J3.4 and J3.5, pages 16.1-122 to 16.1-124)

Vertical Edge Distance,

$Lev = 1.25 \cdot in$

$Le_{min} = 1 \cdot in$

$C_2 = 0 \cdot in$

$Lev_{min} := Le_{min} + C_2$

$Lev_{min} = 1 \cdot in$

$Lev_{max} := \min(6in, 12 \cdot t_{ca})$

$Lev_{max} = 4.500 \cdot in$

RESULT = Lev > Lev.min & Lev < Lev.max, OK

Horizontal Edge Distance,

$Leh_{cab} = 1.5 \cdot in$

$Leh_{cac} = 1.428 \cdot in$

$Leh_{bm} = 2 \cdot in$

$Le_{min} = 1 \cdot in$

$Leh_{mincab} = 1 \cdot in$

$Leh_{mincac} = 1.125 \cdot in$

$Leh_{minbm} = 1 \cdot in$

$Leh_{maxcab} := \min(6in, 12 \cdot t_{ca})$

$Leh_{maxcab} = 4.500 \cdot in$

$Leh_{maxcac} := \min(6in, 12 \cdot t_{ca})$

$Leh_{maxcac} = 4.500 \cdot in$

$Leh_{maxbm} := \min(6in, 12 \cdot tw_{bm})$

$Leh_{maxbm} = 4.260 \cdot in$

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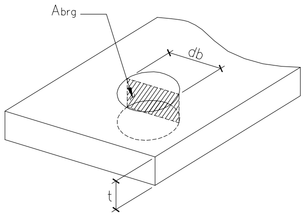
RESULT = Leh > Leh.min & Leh < Leh.max, OK

C. CLIP ANGLE CHECK

1. Bolt Bearing Capacity on Clip Angle

(AISC 14th Ed. Specifications Chapter J, Section J3.10, pages 16.1-127 to 16.1-128)

Bearing Area,



$Abrg_{ca} := db \cdot t_{ca}$

$Abrg_{ca} = 0.281 \cdot in^2$

Beam Side:

Allowable Bearing Strength using edge distance, (J3-6a, J3-6c)

$$Fbe := \Lambda_{brg} \cdot Fu_{ca} \cdot \begin{cases} \min\left[1.0 \cdot \left(Lev - 0.5 \cdot hd_{bcav}\right) \cdot t_{ca}, 2.0 \cdot Abrg_{ca}\right] & \text{if } hd_{bcah} \geq hd_{ls} \\ \min\left[1.2 \cdot \left(Lev - 0.5 \cdot hd_{bcav}\right) \cdot t_{ca}, 2.4 \cdot Abrg_{ca}\right] & \text{otherwise} \end{cases}$$

$Fbe = 10.603 \cdot kips$

Allowable Bearing Strength using bolt spacing, (J3-6a, J3-6c)

$$Fbs := \Lambda_{brg} \cdot Fu_{ca} \cdot \begin{cases} \min\left[1.0 \cdot \left(s - hd_{bcav}\right) \cdot t_{ca}, 2.0 \cdot Abrg_{ca}\right] & \text{if } hd_{bcah} \geq hd_{ls} \\ \min\left[1.2 \cdot \left(s - hd_{bcav}\right) \cdot t_{ca}, 2.4 \cdot Abrg_{ca}\right] & \text{otherwise} \end{cases}$$

$Fbs = 19.575 \cdot kips$

Bolt Bearing Capacity,

$Rbrg_{ca1} := n_{ca} \cdot nv \cdot [\min(Fbe, \Lambda rv) + \min(Fbs, \Lambda rv) \cdot (nr - 1)]$

$Rbrg_{ca1} = 116.632 \cdot kips$

Column Side:

Allowable Bearing Strength using edge distance, (J3-6a, J3-6c)

$$Fbe := \Lambda_{brg} \cdot Fu_{ca} \cdot \begin{cases} \min\left[1.0 \cdot \left(Lev - 0.5 \cdot hd_{ccav}\right) \cdot t_{ca}, 2.0 \cdot Abrg_{ca}\right] & \text{if } hd_{ccah} \geq hd_{ls} \\ \min\left[1.2 \cdot \left(Lev - 0.5 \cdot hd_{ccav}\right) \cdot t_{ca}, 2.4 \cdot Abrg_{ca}\right] & \text{otherwise} \end{cases}$$

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$Fbe = 10.603 \cdot kips$

Allowable Bearing Strength using bolt spacing, (J3-6a, J3-6c)

$$Fbs := \Lambda_{brg} \cdot Fu_{ca} \cdot \begin{cases} \min\left[1.0 \cdot (s - hd_{ccav}) \cdot t_{ca}, 2.0 \cdot Abrg_{ca}\right] & \text{if } hd_{ccah} \geq hd_{ls} \\ \min\left[1.2 \cdot (s - hd_{ccav}) \cdot t_{ca}, 2.4 \cdot Abrg_{ca}\right] & \text{otherwise} \end{cases}$$

$Fbs = 19.575 \cdot kips$

Bolt Bearing Capacity,

$$Rbrg_{ca2} := n_{ca} \cdot nv \cdot [\min(Fbe, \Lambda rv) + \min(Fbs, \Lambda rv) \cdot (nr - 1)]$$

$Rbrg_{ca2} = 116.632 \cdot kips$

Governing Bolt Bearing Capacity,

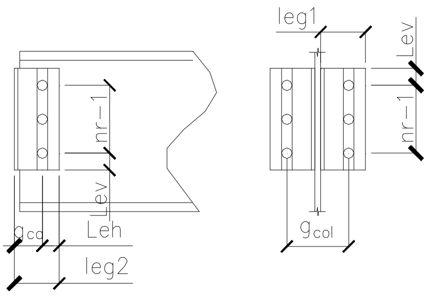
$$Rbrg_{ca} := \min(Rbrg_{ca1}, Rbrg_{ca2})$$

$Rbrg_{ca} = 116.632 \cdot kips$ $V = 100.8 \cdot kips$

RESULT = Bearing Capacity > Force Applied, OK

2. Shear Yielding Capacity of Clip Angle

(AISC 14th Ed, Specifications Chapter J, Section J4.2, page 16.1-129)



Length of Angle,

$$Lca := (nr - 1) \cdot s + 2 \cdot Lev$$

$Lca = 14.5 \cdot in$

Check if Length of Angle is acceptable per AISC requirements,

(AISC 14th Ed. Manual Part 10, page 10-9)

$$Length := \begin{cases} \text{"Angle Length is OK per AISC Requirements"} & \text{if } Lca \geq 0.5(d_{bm} - 2k_{bm}) \\ \text{"Increase Angle Length per AISC Requirements"} & \text{otherwise} \end{cases}$$

Length = "Angle Length is OK per AISC Requirements"

Gross Shear Capacity (J4-3)

$$Rvy_{ca} := \Lambda_{vy} \cdot 0.6 \cdot Fy_{ca} \cdot t_{ca} \cdot Lca \cdot n_{ca}$$

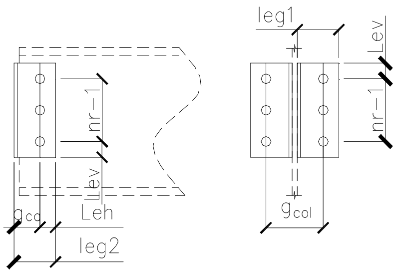
$Rvy_{ca} = 156.6 \cdot kips$ $\qquad\qquad$ $V = 100.8 \cdot kips$

RESULT = Shear Yielding Capacity > Force Applied,OK

3. Shear Rupture Capacity of Clip Angle

(AISC 14th Ed. Specifications Chapter J, Section J4.2, page 16.1-129)

Net Shear Area



$$Anv := \left(Lca - nr \cdot max\left(hd_{bcav}, hd_{ccav}\right)\right) \cdot t_{ca}$$

$Anv = 3.797 \cdot in^2$

Shear Rupture Capacity (J4-4)

$$Rvr_{ca} := \Lambda_{vr} \cdot n_{ca} \cdot 0.6 \cdot Fu_{ca} \cdot Anv$$

$Rvr_{ca} = 132.131 \cdot kips$ $\qquad\qquad$ $V = 100.8 \cdot kips$

RESULT = Shear Rupture Capacity > Force Applied, OK

4. Block Shear Capacity of Clip Angle

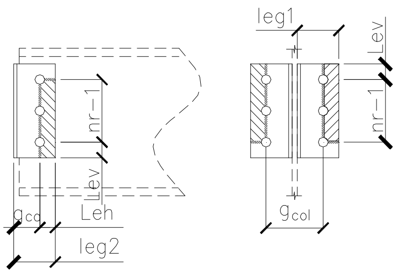
(AISC 14th Ed. Specs. Chapter J, Section J4.3, page 16.1-129)

Reduction Factor, $U_{bs} := \begin{cases} 1.0 & \text{if } nv = 1 \\ 0.5 & \text{if } nv > 1 \end{cases}$

(tension stress is uniform)
(tension stress is non-uniform)

$U_{bs} = 1$

Column/Support Side:



Gross Shear Area

$$Agv := n_{ca} \cdot [(nr - 1) \cdot s + Lev] \cdot t_{ca}$$

$Agv = 9.937 \cdot in^2$

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Net Tension Area

$$Ant := n_{ca} \cdot \left[leg1_{ca} - g_{cac} - (nv - 0.5) \cdot hd_{ccah}\right] \cdot t_{ca}$$

$$Ant = 0.672 \cdot in^2$$

Net Shear Area

$$Anv := Agv - n_{ca} \cdot (nr - 0.5) \cdot hd_{ccav} \cdot t_{ca}$$

$$Anv = 6.984 \cdot in^2$$

Block Shear Capacity on column side,

$$Rbs_{ca1} := \Lambda_{bs} \min\left(0.6 \cdot Fu_{ca} \cdot Anv + U_{bs} \cdot Fu_{ca} \cdot Ant , 0.6 \cdot Fy_{ca} \cdot Agv + U_{bs} \cdot Fu_{ca} \cdot Ant\right)$$

$$Rbs_{ca1} = 126.818 \cdot kips$$

Beam Side

Gross Shear Area

$$Agv := n_{ca} \cdot [(nr - 1) \cdot s + Lev] \cdot t_{ca}$$

$$Agv = 9.937 \cdot in^2$$

Net Tension Area

$$Ant := n_{ca} \cdot \left[leg2_{ca} - g_{cab} - (nv - 0.5) \cdot hd_{bcah}\right] \cdot t_{ca}$$

$$Ant = 0.797 \cdot in^2$$

Net Shear Area

$$Anv := Agv - n_{ca} \cdot (nr - 0.5) \cdot hd_{bcav} \cdot t_{ca}$$

$$Anv = 6.984 \cdot in^2$$

Block Shear Capacity on beam side,

$$Rbs_{ca2} := \Lambda_{bs} \min\left(0.6 \cdot Fu_{ca} \cdot Anv + U_{bs} \cdot Fu_{ca} \cdot Ant , 0.6 \cdot Fy_{ca} \cdot Agv + U_{bs} \cdot Fu_{ca} \cdot Ant\right)$$

$$Rbs_{ca2} = 130.434 \cdot kips$$

Governing Block Shear Capacity of Clip Angle,

$Rbs_{ca} := \min(Rbs_{ca1}, Rbs_{ca2})$

$Rbs_{ca} = 126.818 \cdot kips$ $V = 100.8 \cdot kips$

RESULT = Block Shear Capacity > Force Applied, OK

D. CLIP ANGLE TO COLUMN CHECK

1. Bolt Shear Capacity

(AISC 14th Ed. Specifications Chapter J, Section J3.6, page 16.1-125)

Shear Capacity per Bolt

$\Lambda rv = 11.928 \cdot kips$

Bolt Shear Capacity,

$Rb_{v2} := n_{ca}\, n \cdot \Lambda rv$

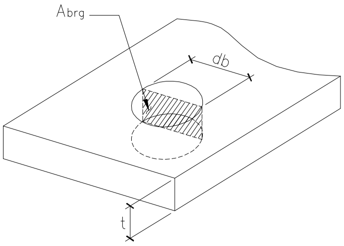
$Rb_{v2} = 119.282 \cdot kips$ $V = 100.8 \cdot kips$

RESULT = Bolt Shear Capacity > Force Applied,OK

E. COLUMN CHECK

1. Bolt Bearing Capacity on Column

(AISC 14th Ed. Specifications Chapter J, Section J3.10, pages 16.1-127 to 16.1-128)



Bearing Area,

$Abrg_{col} := db \cdot tf_{col}$

$Abrg_{col} = 0.675 \cdot in^2$

Allowable Bearing Strength using edge distance, (J3-6a, J3-6c)

$$Fbe := \Lambda_{brg} \cdot Fu_{col} \cdot \begin{cases} 2.0 \cdot Abrg_{col} & \text{if } hd_{col} \ge hd_{ls} \\ 2.4 \cdot Abrg_{col} & \text{otherwise} \end{cases}$$

$Fbe = 52.65 \cdot kips$

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Allowable Bearing Strength using bolt spacing, (J3-6a, J3-6c)

$$Fbs := \Lambda_{brg} \cdot Fu_{col} \cdot \begin{cases} \min\left[1.0 \cdot \left(s - hd_{col}\right) \cdot tf_{col}, 2.0 \cdot Abrg_{col}\right] & \text{if } hd_{col} \geq hd_{ls} \\ \min\left[1.2 \cdot \left(s - hd_{col}\right) \cdot tf_{col}, 2.4 \cdot Abrg_{col}\right] & \text{otherwise} \end{cases}$$

$$Fbs = 52.65 \cdot kips$$

Bolt Bearing Capacity,

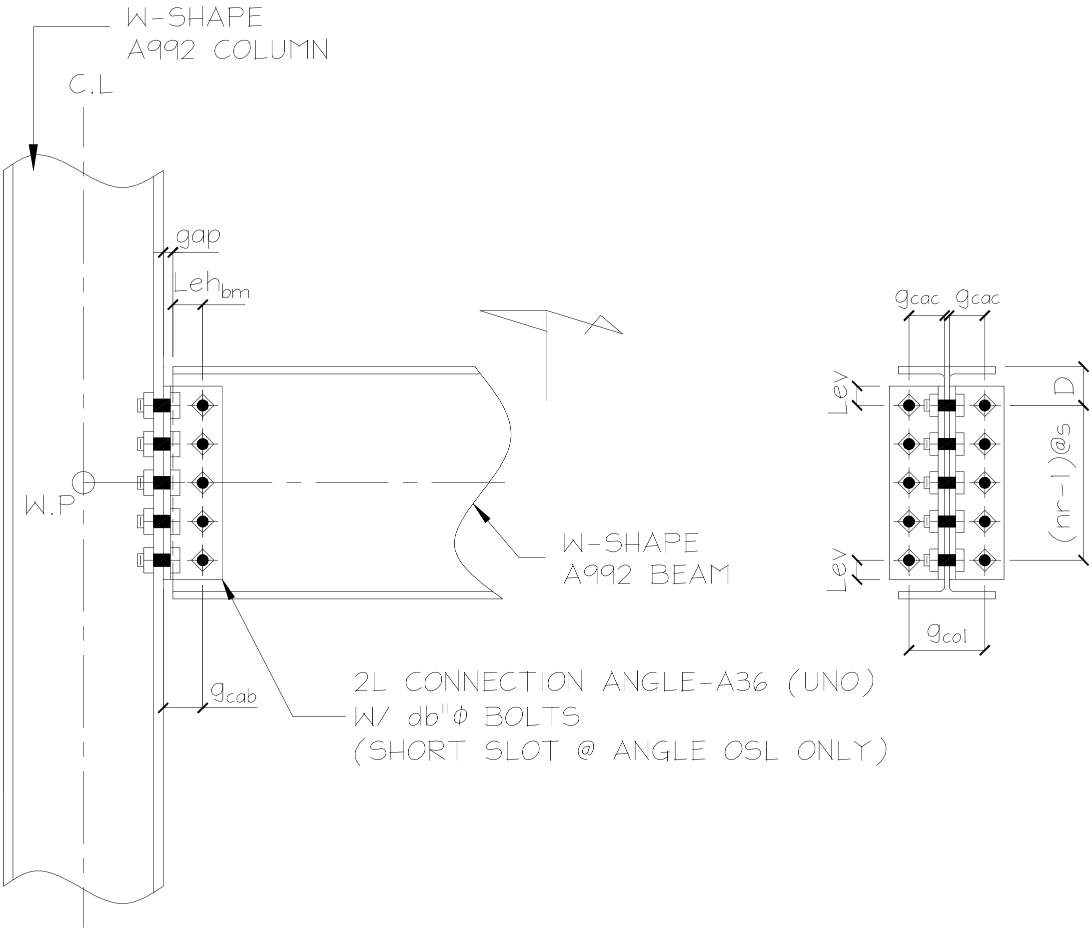
$$Rbrg_{col} := n_{ca} nv \cdot [\min(Fbe, \Lambda rv) + \min(Fbs, \Lambda rv) \cdot (nr - 1)]$$

$Rbrg_{col} = 119.282 \cdot kips$ $V = 100.8 \cdot kips$

RESULT = Bearing Capacity > Force Applied,OK

III. DETAILS

A. SKETCH



NOTE: (FIGURE ABOVE DOES NOT REPRESENT ACTUAL DESIGN, REFER ON ATTACHED CONNECTION SCHEDULE FOR NUMBER OF BOLTS)

SHEAR CONNECTION: DETAIL OF W-SHAPE BEAM TO W-SHAPE COLUMN FLANGE CLIP ANGLE CONNECTION (BOLTED-BOLTED)

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B. TABLE: SHEAR CONNECTION SCHEDULE

Column		Beam		D (in)	g_{col} (in)	gap (in)	Beam Shear Load (kips)	Rcap (kips)	Governing Capacity
Size	Grade	Size	Grade						
W12X96	A992	W18X50	A992	3	5 1/2	1/2	100.8	103.8	Bolt Bearing on Beam

Connection Angle					Bolt Type	Bolts		s (in)	Lev (in)
Size	Grade	g_{cab} (in)	g_{cac} (in)	$leg2_{ca}$ (in)		db (in)	nr		
2L4X4X3/8	A36	2 1/2	2 4/7	4	A325-N	3/4	5	3	1 1/4

IV. REFERENCES

Steel Construction Manual (14th)- ASD American Institute of Steel Construction, Inc. 2010